Lehigh University

INSTITUTE OF RESEARCH

Welded Continuous Frames & Their Components

Progress Report U

THE COLLAPSE STRENGTH OF A WELDED SINGLE BAY FRAME

by

F. W. SCHUTZ, JR., C. G. SCHILLING and L. S. BEEDLE

(Not for Publication)

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This work has been carried out as part of an investigation sponsored jointly by the Welding Research Council of the Engineering Foundation and the Navy Department with funds furnished by the following:

American Institute of Steel Construction
American Iron and Steel Institute
Office of Naval Research (Contract No. 39303)
Bureau of Ships
Bureau of Yards and Docks

Fritz Engineering Laboratory
Department of Civil Engineering and Mechanics
Lehigh University
Bethlehem, Penna.

August 1953

Fritz Laboratory Report No. 205D.5

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I. INTRODUCTION

1. OBJECT AND SCOPE OF INVESTIGATION

The test reported herein is the third of several to be carried out at Lehigh University as part of the broad investigation titled "Welded Continuous Frames and Their Components". The frame tested was a simple portal frame statically indeterminate to the first degree and was fabricated from 12WF36 with a beam span of 30 ft. and a column height of 10 ft. Previous tests had been carried out on two similar portal frames with 14 ft. beam spans and 7 ft. column heights. These earlier frames were formed from 8WF40 and 8B13 shapes. Theretofore, all frames had been tested under vertical loads only but the present test frame was subjected to horizontal and vertical loads.

The test was planned so that it would simulate the action of a full size portal frame subjected to a ratio of vertical load to horizontal load as might be expected in a severe wind storm. Great care was taken to insure proper lateral support and to measure the forces exerted by the lateral supports. Deformations were measured at various critical locations in the frame in order to compare its behavior with the theoretical analysis.

II. DESCRIPTION OF FRAME AND TESTING APPARATUS

2. TEST SPECIMEN

The test specimen used is detailed on Fig. 1. It is a full-sized single bay rectangular rigid frame fabricated from a 12WF36 steel section. The knees for the frame are of type 8B described in Progress Report 4.(1)* The column bases were mounted on knife edges so that a pin ended condition was maintained throughout the test. The distance between the column bases was kept constant by means of tie rods.

The beam span was 30 ft. and the column height was 10 ft. The loads were applied at the one third points of both the beam and windward column. The frame was taken from an imaginary building in which the frame spacing was 15 ft. and the vertical working load was taken as 60 psf and a design wind load of 20 psf. This combination gives for the particular frame dimensions a total vertical load nine times as large as the total horizontal load. This ratio of vertical load to horizontal was maintained throughout the test.

The steel section used in fabricating the portal frame was a nominal 12WF36 as mentioned above but the actual measurements of the cross-section showed that the section used had properties that varied to some extent from those given in the A.I.S.C. Steel Construction Manual. A comparison of handbook and actual dimensions is given in Table I. From this comparison one can see that the cause of most of the discrepancies is the difference between the actual and handbook values for flange thickness. The resulting variation in the section modulus, S_x, and plastic modulus, Z_x,

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* Numbers in parentheses indicate the reference number in the bibliography.

cause the yield moment and plastic moment to be lower than the handbook values by 6.4 and 7.1 per cent, respectively.

TABLE I PROPERTIES OF 12WF36

	Wgt. per ft. lb.	Area in^2	Depth in.	Flange Width in.	Flange Thick. in.	Web Thick. in.	I_x in^4	S_x in^3	Z_x in^3
Actual	34.4	10.13	12.17	6.560	0.506	0.306	264	43.5	48.1
Handbook	36	10.59	12.24	6.565	0.540	0.305	280.8	45.9	51.5
Variation	-4.6%	-4.6%	-0.6%	-0.1%	-6.7%	+0.3%	-6.4%	-5.5%	-7.1%

The mechanical properties of the steel used were determined by standard coupon tests (both tension and compression) taken from several locations in the cross-section of the beam. The steel used was ordered to meet the requirement of ASTM Designation A7-50T and all three pieces needed to form the frame were cut from a single length.

The mill report for the steel is shown in Table II.

TABLE II MILL REPORT ON 12WF36

Chemical Composition in Per Cent:	Mechanical Properties:
C = 0.18	Yield Strength (upper yield) = 42,530 psi (Avg. Yield Stress Level by Laboratory Tests = 39,100 psi)
Mn = 0.65	
P = 0.014	Ultimate Strength = 67,420 psi
S = 0.038	Elongation in 8 in. = 25.2 per cent
	Reduction in Area = 50.0 per cent

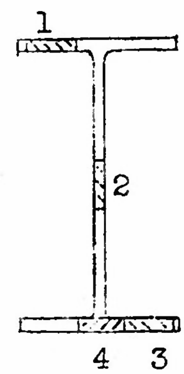
The laboratory coupon tests are summarized in Table III. In using these results the yield stress level of those coupons (tension and compression) located in the flanges of the beam were averaged and used to determine the yield moment and plastic moment

of the section. This average yield stress level was 39,100 psi which is somewhat lower than the upper yield strength of 42,530 psi given in the mill report.

TABLE III SUMMARY OF LABORATORY

COUPON TESTS OF 12WF36

Location	Tension or Compression	Yield Stress Level psi	Ultimate Strength psi	Strain Hardening ϵ_s in/in
1	T	39,230	62,000	.015
1	C	38,060	-	.014
2	T	45,100	67,800	.024
2	C	45,150	-	.014
3	T	39,700	62,200	.018
3	C	38,090	-	.015
4	T	41,200	66,200	.014
4	C	38,490	-	.013



3. LATERAL SUPPORT

Past experience in the testing of rigid frames into the plastic region had shown that adequate lateral support was essential if the theoretical collapse load where to be attained. Therefore, the present test frame was provided with a lateral support system which might be equivalent to that used in actual building construction. This support was given by 18 struts which constrained the frame to deform in a plane about 10 ft. from the wall of laboratory building. The locations of the 18 lateral support struts are indicated by the small circles drawn on the flanges of the beam on Fig. 1. A numbering system for the lateral supports is also indicated on the drawing.

In order to insure free movement of the frame in its plane, the lateral support struts were fitted with flex bars at each end. SR-4 electrical strain gages were attached to one of the flex bars of each lateral support strut so that the force in the individual struts could be ascertained at any time.

The lateral support system may be seen in the photograph of the general test arrangement shown in Fig. 2.

4. LOADING SYSTEM

The loads were applied to the frame by means of hydraulic jacks. Four jacks were used in all, one jack for each of the two vertical loads, one for the horizontal load and one for the horizontal reaction at the base of the windward column. An aluminum tube dynamometer was used in conjunction with each jack. The loading system may be seen in Fig. 2 and 3. All loads were applied to the frame through a horizontal pin located at the centroid of the beam cross-section. Transverse stiffener plates were used to help distribute the load to the beam at these points.

In order that the minimum amount of adjustment would have to be made to the lateral support system, the test was planned so that the movement caused by the horizontal load would take place at the column bases leaving the ends of the beam more or less fixed in space. Fig. 3 shows how the column bases and horizontal loading system was arranged so that this movement could take place.

The tie rods used to maintain the distance between the column bases were connected in series with aluminum bar dynamometers allowing the tie rod force to be measured.

5. ROTATION MEASUREMENTS

Measurements to determine the rotation occurring along a unit length of beam and across the knees of the frame were made by use of the rotation indicators described in Progress Report 7[2] and illustrated on Fig. 20 of that report. Four such rotation indicators were used on the present frame, one across each knee, one on the beam near the point where the second plastic hinge formed and one on the leeward column where the first plastic hinge formed. These indicators may be seen in the photographs of Fig. 2 and 3.

6. FLANGE DISPLACEMENT MEASUREMENTS

The movement of the beam flanges with respect to one another was measured by a mechanical micrometer dial used as indicated by Fig. 17 of Progress Report 7.[2] Measurements were made with the dials at 6 locations in the regions of the plastic hinges. Further evidence of flange crippling was obtained by pairs of SR-4 electrical strain gages mounted on opposite sides of the compression flange in the plastic hinge zones.

7. DEFLECTION MEASUREMENTS

Ordinary surveying instruments were used to determine the deflected shape of the frame to within 1/50 of an inch. The accuracy of the deflections obtained was adequate in view of the fact that deflections of 3 inches were obtained at collapse load, and these increased to 10 inches at failure. Two transits were set up on the laboratory floor near each frame column and their telescopes were oriented, and maintained, in fixed vertical planes

perpendicular to the plane of the frame. By sighting on a scale held at right angles to the column, the distance from this fixed vertical plane to the point on the column could be ascertained.

The distance from the beam of the frame to a fixed horizontal plane of sight was determined in a similar fashion by a level mounted on the balcony of the laboratory so that its line of sight was just above the undeformed position of the top flange of the beam.

The system of measuring deflections described above had very distinct advantages over methods used on previous frame tests. There was no rig or frame mounted on the test frame to interfere with photography or the reading of other deformation measuring devices. The need for several adjustments of the deflection measuring instruments during the test was eliminated.

8. TEST PROCEDURE

The test procedure used on the frame took the form of 3 phases as follows:

1. Check test of the frame as a determinate structure in elastic range.
2. Test of the frame as an indeterminate structure in the elastic range.
3. Main test through the elastic and plastic range to final failure by lateral plastic buckling of the lee column.

To be sure that the testing apparatus was working according to plan and to check on the action of the test frame, the tie rods between the pinned bases were removed making the frame statically determinate. In this condition the frame was loaded in 3 different ways and the resulting deflections measured. First

only the 2 vertical loads were applied and the resulting horizontal movements of the bases measured as well as the beam centerline deflection. The same deformations were measured in the subsequent check runs during which loads were applied to the windward column and then to the column bases. These check test results had a maximum variation of 6.5 per cent from the theoretical values indicating the testing apparatus and frame were behaving in a satisfactory manner.

With the tie rods in place so that the distance between the columns bases was maintained constant at all times, a test run was made of the structure as a statically indeterminate frame in the elastic range. This check also showed that the test set up was performing in good fashion as indicated by the fact that the measured tie rod force agreed within 2 per cent with the force indicated by elastic analysis.

With assurance that the test equipment was performing as planned, the main test was started and carried out continuously for 60 hours until lateral plastic buckling occurred in the lee column.

During the early stages of the test, readings were made on all measuring equipment at frequent load intervals. No data was taken at a given load increment until the centerline deflection of the beam had shown that no significant change in deformation would occur if the load were held constant for a longer period of time.

As the applied loads approached the theoretical plastic collapse load the time required for the deformation of the structure to reach a constant level under the constant load became longer and longer. To overcome this long wait for the frame to

"settle down" a "deformation-increment" criterion was adopted for the remainder of the test. During this latter part of the test no data was taken at a given deformation increment until the load had shown no significant change in load would occur if the deformation were held constant for a longer period of time. This meant that after a set of reading had been completed an increment of deflection for the beam center was chosen to be added to the existing deflection. The frame was then deformed this amount by pumping on the jacks being careful to maintain the proper ratio of loads in all jacks at the same time. Once the proper deformation level had been reached, it was held constant while observations of the load variation with respect to time were made. When the loads showed no tendency to change at the constant deformation level, it was assumed the frame had "settled down" and a complete set of data was taken.

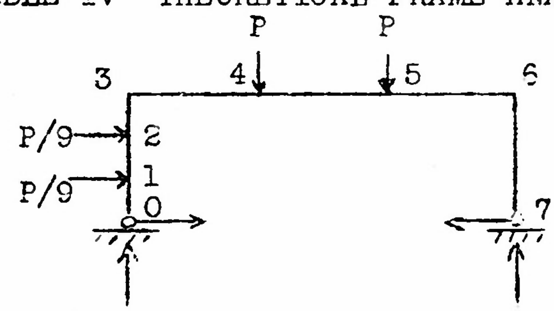
The above described "deformation-increment" criterion for determining when a set of data might be taken when the structure was in the plastic range proved to be far less time consuming than the "load-increment" method used on earlier frame tests.

III. THEORETICAL ANALYSIS

9. LOADS, REACTIONS AND MOMENTS

A very simple theoretical analysis will be presented here to indicate the predicted behavior of the frame. When the loads are such that no part of the frame must endure strains that are above the yield strain, the structure may be investigated by the ordinary elastic analysis method for indeterminate structures. Such an analysis will give the moments for the various critical points of the frame shown in Table IV. (These moments are also shown in Fig. 5.)

TABLE IV THEORETICAL FRAME ANALYSIS



Condition of Frame (1)		All Elastic (2)	At First Yield (3)	At Collapse by Simple Plastic Theory (4)
Vertical Load		P	23.5	29.2
Horizontal Reactions at Pts. in Kips	0	0.382 P	9.95	9.39
	7	0.604 P	14.17	15.67
Vertical Reactions at Pts. in Kips	0	0.963 P	22.6	28.1
	7	1.037 P	24.4	30.3
Moments at Pts. in In. Kips	1	15.3 P	358	367
	2	35.0 P	820	864
	3	59.2 P	1390	1490
	4	56.7 P	1333	1880
	5	52.4 P	1230	1760
	6	72.5 P	1700	1880

By using the actual section modulus of 43.5 in^3 for the wide flange section and using the average yield stress level of 39,100 psi, the yield moment, M_y, for the frame is found.

$$M_y = S_x \times \sigma_y = 43.5 \times 39.1 = 1700 \text{ In. Kips}$$

From the elastic analysis it is noted that the maximum elastic moment occurs at point 6 (see Table IV) where the moment is 72.5 P. Thus a theoretical yield load, P_y, of 23.5 kips at each of the beam load points is established.

The elastic analysis shows that the first plastic hinge must form at point 6 (the leeward knee of the frame). Only one more hinge is needed to allow the frame to collapse. This second hinge will form at point 4 (the beam load point nearest the windward column). With the plastic modulus known (Z_x = 48.1 in^3) and using the yield strength from coupon tests of the flange (σ_y = 39,100 psi) the plastic moment, M_p, can be computed.

$$M_p = Z_x \times \sigma_y = 48.1 \times 39.1 = 1880 \text{ In. Kips}$$

With the moments at points 4 and 6 known, it becomes a matter of statics to determine the load, reactions and moments at other points of the frame. These values are listed in Table IV in column 4.

10. DEFORMATIONS

In order to check the actual behavior of the frame against the theoretical behavior, some measurable quantity other than applied forces should be predicted by theoretical means. One such quantity chosen for the present test is the deflection of the center of the beam.

While the frame is in the elastic range the beam center-line deflection may be determined by ordinary elastic analysis. However, such analysis assumes the frame to be formed from members having lengths given by the centerline dimensions of the frame. This assumption leads to an answer which is approximately correct but it can be improved upon by taking into account the fact that the particular knees of the frame rotate more than the equivalent length of plain beam. A rational method of predicting such difference in rotation is given in Progress Report 4.(1)

Once the increased rotation of the knee is known the added deflection of the beam due to the knee flexibility at some specific point, say at the center of span, may be computed by solving the case of a simple beam which has slopes at its ends the same as the increase in knee rotation over an equivalent length of plain beam. In the case of the present frame the increase in the deflection in the center of the beam due to knee rotation was only 0.05 in. at the yield load compared to a deflection of 1.74 in. given by the usual methods of elastic analysis. Thus the corrected theoretical deflection at yield load, 23.5 kips, is 1.79 in.

Approximate values for deflections of the frame may be determined just as the collapse is reached by a very simple method described by Symonds (3) and in Progress Report No. 3.(4) This method assumed that yielding is concentrated at the plastic hinges and that these hinges are free to rotate under the constant moment, M_p, other parts of the frame remaining elastic. Just as the last plastic hinge is formed the slope at either side of the hinge must be equal. Using these assumptions and the well known slope deflection equations, one may find the deflected shape of the struc-

ture. For the present test frame this gives an estimated center beam deflection of 2.82 in. at collapse load. Again the computed deflection should be increased because of the knee flexibility. For lack of a better method the amount of additional deflection due to the added knee rotation in the knee will be taken as value from the elastic case at load P_y multipied by P_p/P_y. This gives a value of 0.06 in. Thus the deflection of the beam center at ultimate load becomes approximately 2.87 in. It seems reasonable that this deflection should be larger since yielding is spread out over lengths of the beam and not concentrated at the hinges as assumed in the analysis. This would be particularly true in the present case since much of the center third of the beam is withstanding moments greater than M_y when the collapse load is reached. Hence the actual deflection at ultimate load should be somewhat larger than the value predicted above.

The deflection computations discussed above allows one to draw the theoretical load deflection curve shown in Fig. 4. The theoretical curves for the elastic moment-unit rotation relationship for plain beam sections were obtained from the basic relation ϕ equals Moment divided EI. The theoretical moment-rotation relationship for the knees in the elastic range was found by use of equations developed by Beedle.[5]

IV. RESULTS OF TESTS

11. GENERAL BEHAVIOR

The present test frame and test apparatus behaved as well or better than was expected in all respects. It is believed that the results indicate the performance that might be expected from an actual building frame where the proper consideration is given the lateral support system. At the same time, a lateral support system capable of providing the support given the test frame might not be impractical; in fact, even better support might be provided in an actual building.

The frame carried the predicted yield load and collapse load with deformations which were very close to the predicted values. In addition, the frame showed an ability to carry the predicted collapse load even when the deflections were double those at the time the collapse load was first reached.

Failure was brought about when the lee column buckled laterally. This buckling occurred in a region that was fully plastic and was a clear case of plastic instability. Other minor cases of plastic instability took place but were prevented from progressing to such an extent as to be the cause of the frame failure. The ability of the frame to survive these earlier cases of plastic instability was undoubtedly due to the effective lateral support system.

The load carrying capacity of the frame over and above that predicted by normal elastic theory is illustrated in one way by the moment diagrams drawn on Fig. 5. The diagrams drawn are for two cases of actual moment and for two cases of theoretical moments.

One of the theoretical moment diagrams shown is for elastic limit condition and the other is found for the collapse load condition. The actual moments were computed from measured forces and were corrected for the frame deformations. The moment diagram that existed when the nominal maximum stress was 20 ksi is shown by the solid line. The dashed line shows the theoretical elastic limit case.

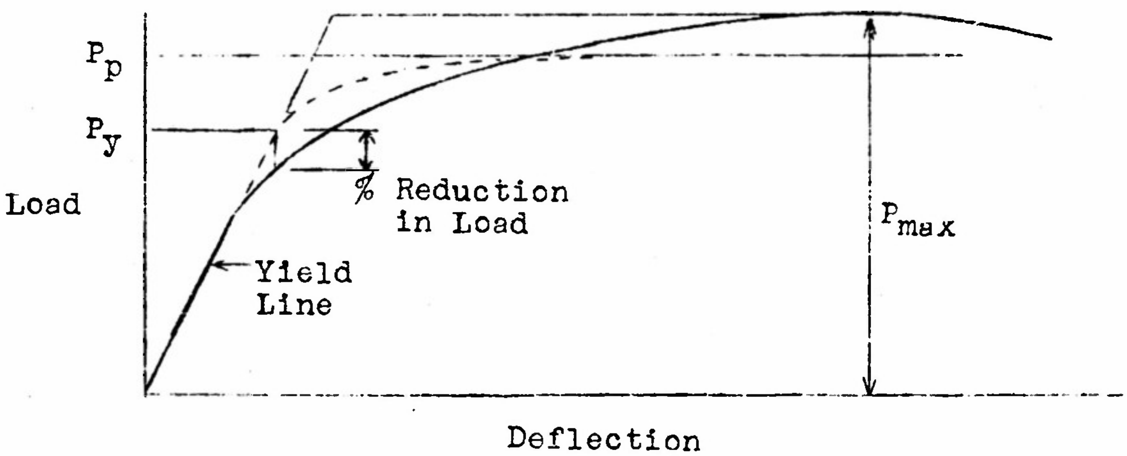
The other 2 curves do not vary from one another much and indicate how well the frame was in accord with simple plastic theory. These 2 curves represent the predicted moments at collapse load by simple plastic theory and the ultimate load moments. After the ultimate load condition had been reached, the moments for all loadings prior to the lee column buckling were nearly identical to the moments at ultimate load.

12. LOAD-DEFLECTION RESULTS

With regard to load carrying capacity the test results more than met the behavior predicted by simple plastic theory. The frame's response to loads in the elastic range was also very good. In fact, compared to the action of the two previous frames, the present frame showed near perfect agreement between observed and computed values. Table V compares the test results with the predicted behavior. The result of the previous frame tests at Lehigh University are shown on the same table for convenience.

TABLE V STRENGTH COMPARISON

Frame No.		Yield Strength			Maximum Strength	
		First Yield Line kips	General Yield kips	Load Reduction at P_y	Elastic Analysis Comparison kips	Plastic Analysis Comparison kips
1 (8WF40)	Observed	22.0	40.4		52.4	52.4
	Computed	39.4	39.4		39.4	47.7
	Ratio	0.56	1.05	16.5	1.33	1.10
2 (8B13)	Observed	5.5	12.2		18.0	18.0
	Computed	13.1	13.1		13.1	18.1
	Ratio	0.42	1.01	11.5	1.37	0.99
3 (12WF36) Present Frame	Observed	15.9	26.6		29.7	29.7
	Computed	23.5	23.5		23.5	29.2
	Ratio	0.68	1.13	6.2	1.26	1.02



Further evidence of the close agreement between theoretical and test behavior is given on Fig. 4 where the load-deflection curves are shown. The actual deflection at theoretical collapse load is 12 per cent larger than the computed value.

The actual ultimate load of 29.7 kips in each vertical jack is 1.02 times greater than the predicted collapse load of 29.2 kips. Of particular interest is the fact that the frame continued to carry loads equal to the computed collapse load even when the deflection had increased to twice the deflection computed for ul-

timate load. This curve shows that the frame has a static energy absorbing capacity some 3 times greater than the energy required to reach collapse load and still has the ability to support a load equal to the computed collapse load.

Another presentation of the manner in which the structure deformed is given in Fig. 6. There the deflected shapes of the frame at several load conditions are shown. The first deflection curve drawn shows the shape of the frame when loaded with a vertical load in each jack of 12 kips. This load produces a moment of 870 in. kips at the lee knee and a unit stress of 20,000 psi. It is equal, then, to a normal design load by conventional elastic methods.

The second deflected shape of Fig. 6 is drawn for a vertical load of 18 kips. This load is 1/1.65 of the ultimate load and might be the maximum design under a plastic analysis method. At this load the frame is still well within the elastic limit. The maximum deflection at this load was 1.47 times the maximum deflection at the normal design load.

The shape of the frame at ultimate load, 29.7 kips, is given by the third curve on Fig. 6. This curve shows that the maximum deflection at the ultimate load is only 3.5 times as great as for the normal elastic design load.

The curve having the largest deviations is for the last load put on the structure and represents the greatest deformation that occurred. The load at this time was 26.5 kips. The lee column had already buckled laterally at this stage of the test. Despite the column failure and the large distortions the frame was still carrying 89 per cent of the ultimate, 221 per cent of the normal elastic design load, and 147 per cent of a possible plastic

design load which uses a safety factor 1.65 against ultimate. A photograph of the frame after testing is shown in Fig. 7. The deformed structure shown in Fig. 7 closely approximates the shape shown by the maximum deflection curve shown in Fig. 6. Comparison of these two figures indicates the magnification given the deflections in Fig. 6 where the deflections are plotted on a scale 4.8 times larger than the scale used to lay out the frame dimensions.

13. MOMENT-ROTATION RELATIONSHIPS

Since one of the basic requirements of a material and a section to be used in a structure designed by plastic analysis is the ability to form plastic hinges, it is of interest to study the moment-rotation relationship of certain critical parts of the test frame. One such critical part is the knee. The knee design used in this frame had been investigated at Lehigh University in earlier phases of the present program descirbed in Progress Report 4[1] where it is classified as a type 8B connection. The knee details are, to scale, the same as those for Connection L (Type 8B) of Progress Report 4. It had shown good results in these earlier connection tests and was therefore a logical choice to be used in the frame.

The moment-rotation curves for both knees are shown in Fig. 8. At no time during the test did the knees show signs that they had smaller moment capacity than the beam section. There was no local crippling of any parts even though yielding of the material was widespread in the knee at the lee column. The knee showed the capacity to carry the full plastic moment of the beam section through large rotations. The moments at the intersection of beam

and column centerlines based on measured reactions and measured frame deflections are used in the plotting of one set of curves (drawn with solid lines) shown on Fig. 8. The second set of curves (drawn with dashed lines) were plotted from moments not deformed. This difference in these two sets of curves becomes significant only at very large rotations well after the ultimate load had been reached in the frame.

The knee at the windward column was never called upon to carry a moment equal to the theoretical yield moment; nevertheless, the moment-rotation curve for this knee is not a straight line and when the frame was unloaded the knee had taken on a small amount of permanent set, indicating inelastic action. Fig. 8 shows that the two knees behaved in almost identical fashion at equal moment levels.

Even though the simple plastic analysis assumes that the plastic hinges form at points on the frame the actual hinge may be spread over a considerable length of the frame. This is particularly true where the plastic hinge at a knee, where the knee is somewhat stronger than the as-rolled beam section. For the present frame the yield zone near the first plastic hinge at the lee knee was widespread by the time the ultimate load was reached. Fig. 9a shows this region and the extent of the yielding shortly after the ultimate load was reached. The flaking of the whitewash on the frame indicates yielding has occurred. It will be noted that this zone carries further along the column than along the beam, agreeing with the fact that the beam has a much steeper moment gradient than the column (see Fig. 5). The spread of the second plastic hinge at the same load is shown in Fig. 9b. Since the moment curve between the vertical load points is virtually flat, the yielding occurs over

a considerable length of beam. Even so, there is a concentration of yielding near the second plastic hinge (the windward vertical load point).

The moment-unit rotation curve for the section of the column just below the first theoretical plastic knee is shown on Fig. 10. Again the solid line represents the relationship when the distortion of the frame is taken into account when computing the actual moment at the section of the column and a dashed line is for the case where distortion is neglected. These curves show that the full plastic moment was never reached at this portion of the frame; nevertheless, what appears to be plastic hinge action was started at the ultimate load condition when the moment at the section was 96.1 per cent of the theoretical plastic moment. As the rotation increased rapidly after the ultimate load had been reached, the moment increased slightly to 99.7 per cent of the M_p value but only after the rotation was about five times greater than it was at the ultimate load. Previous tests of beams had shown similar lower actual plastic moment values.[6] The lowering of the actual plastic moment has generally been attributed to residual stresses in the beams.[7]

It should be pointed out that the moment carrying capacity at this location was not appreciably decreased until the column buckled laterally. The rotation at which column buckling occurred is indicated by the symbol "L.B." on the figure.

Fig. 11 shows the moment-unit rotation relationship found by the rotation indicator mounted on the frame near the theoretical location of the second plastic hinge. These curves are very similar to the curves shown on Fig. 10 except for the drop in the moment which occurs just after the ultimate was

reached. This reduction can possibly be explained by the fact that the beam tried to buckle laterally in this region soon after ultimate load was reached. This buckle could be observed by eye at the rotation indicated by the symbol "L.B.", but its effect was undoubtedly indicated much sooner by the drop in moment at this section and the drop in applied load seen on Fig. 4. The effect of this lateral buckling action was quickly overcome as the lateral supports in the region were sufficient to prevent increased lateral movement. The moment at the section increased again and exceeded the previous maximum value.

14. PLASTIC BUCKLING AND LATERAL SUPPORT

The present frame showed once again the fact that the final failure of continuous rigid frames is usually brought about by instability of some part or parts of the frame. The proportions of most frames and rolled sections are such that this instability does not develop in the elastic range. Once the steel has yielded, however, the possibility of this phenomenon occurring is increased many times. At the present it is only by such tests as described herein that one is able to find out with any degree of certainty whether or not a certain frame, made from a certain beam and loaded in a particular fashion, is able to carry its predicted plastic collapse load before this instability causes final failure.

Since the only way to prevent instability failure is to support the compressed zones of the frame transversely, the location and strength of the lateral support system for a frame becomes of primary importance. At the same time the width to thick-

ness ratio of unsupported outstanding flanges becomes very important, since they may suffer from local flange buckling or flange crippling and thus bring about failure of the frame.

The present test frame suffered from buckling in three regions. All three zones affected were in a plastic state when the buckling occurred.

The first evidence of instability was observed by eye after the ultimate load had been reached and took the form of a lateral displacement of the compression flange of the beam near the second plastic hinge. The effect of this lateral buckle has already been discussed with regard to the drop in moment capacity of the beam in the region where the buckle occurred. This buckle took the form of a wave about 3 ft. long, but its displacement was prevented from increasing by the lateral supports which were attached to the beam at the intersection of web and flange.

At the same time that the lateral buckle was observed in the top flange in the middle third of the beam, another type of instability was observed in the bottom flange of the beam at the lee knee in the form of flange crippling. The buckle occurred only in one-half of the flange with a wave length of about 3 or 4 inches. The center of the wave was about 4 in. from the intersection of beam and column. The yielded zone in which this buckle occurred can be seen in Fig. 9a. The buckle could be seen on the beam at the time the photograph was taken, but it is not easily discernible in the photograph. Though this buckle was observed soon after ultimate load had been reached, it did not appear to hinder the performance of the frame in any way. Certainly it did not have the weakening effect shown by the lateral buckle which occurred in the middle third of the beam.

In this second case of instability as in the first case described, good lateral support was near at hand and may have prevented damage that might have developed had it not been there.

The third case of instability came when the unsupported compression flange of the lee column buckled laterally and the frame failed. This buckle showed some early signs of developing by the unequal yield pattern on the flange but apparently was held in check for some time by the lateral support attached to the compression flange at the intersection of beam and column. However, when the deflection at the center of the beam had reached a value of about 2.3 times its value at ultimate load, there was a distinct and rapid increase in the size of the buckle wave and a corresponding drop in load. However, even after this buckle occurred, the frame supported 87.2 per cent of its ultimate load but further straining produced decreasing load carrying capacity. Just before the lee column buckled the load was 95.3 per cent of the ultimate load and the deflection was 230 per cent of the deflection at ultimate load.

The buckle in the lee column is shown after completion of the test in Fig. 12. The fact that the buckle was of the lateral buckling type is shown in Fig. 12a where the lateral displacement of the compression flange is shown clearly.

The area in which the lateral buckle in middle third of the beam occurred is shown in Fig. 13. The displacement of this buckle was so small that it is not easy to see in these photographs. The photographs do show very well the widespread yielding that had taken place at the second plastic hinge by the time the final loads had been applied.

It has already been pointed out that earlier failure of the frame was undoubtedly prevented by the effective lateral support furnished for the test frame. A study of the forces that were measured in the lateral supports showed that the frame required negligible lateral support in the elastic range, but as zones of yielding in the frame formed, the lateral support system was called upon to carry larger and larger loads. Those lateral support struts at the theoretical plastic hinges were called upon to carry the larger part of the lateral loads. When the frame was at the verge of failure, there was a total of 12,660 lbs. tension and 12,660 lbs. compression in the lateral support struts; at the same time the single forces required at the first and second hinges were 3580 lbs. each. Thus the lateral forces at the plastic hinges made up 57 per cent of the total lateral force.

To obtain a dimensionless plot of the relationship between moment and lateral support forces at the plastic hinges, the moment at the section was divided by the theoretical yield moment and the lateral support force was expressed as a percentage of the resultant of the compressive stresses in the beam. The resultant of the compressive stresses is found by dividing the moment by the distance between the resultant of the tensile and compressive stresses. In the elastic condition the distance between the resultant is obtained by dividing the Moment of Inertia by the first moment of one-half of the section about the neutral axis of the whole section. The distance for the fully plastic case is equal to the plastic moment divided by the product of the yield stress and one-half of the section area. The two distances thus found will be the limit for any other strain condition of the beam.

Such dimensionless plots for the two hinges in the present frame are shown in Fig. 14. From this graph it will be seen that the maximum lateral support force was never more than about 2 per cent of the resultant of the compressive stresses in the beam. In fact the total of all lateral forces was never more than about 7 per cent of the resultant of the compression stresses at the plastic moment for the beam.

In order that the distribution of the forces in the various lateral support struts might be seen for two critical load conditions, the graphs on Fig. 15 were drawn. At the top is drawn the actual moment diagram for the beam of the frame at the load just before the lee column buckled laterally. The moment diagram for the maximum load case was essentially the same and is shown on Fig. 5. The solid bars on the graphs show the force in each lateral support strut at the load just before failure by the lateral buckling of the lee column. The shaded bars represent the same forces at the ultimate load condition. The arrows at the ends of the bars indicate the direction of the forces. If the arrow points down, the particular lateral support strut was in compression.

Several facts illustrated by these plots should be pointed out. The maximum values of the lateral forces occurred at the plastic hinges. The larger lateral forces occur at the compression flange of the beam. The presence of the lateral buckle in the top flange in the middle third of the beam is evident from the large values of lateral load in the two lateral support struts to the right of the windward vertical load point. Virtually no force was required to constrain the windward knee which was never subjected to a moment as large as the yield mo-

ment for the beam section. The forces at the top and bottom of the beam at any one section were always unequal or of opposite sense indicating that a twisting tendency always existed when the plastic condition had been reached.

The presence of flange crippling and its effect on the frame strength has been mentioned briefly above. The subject needs some further comment. First it should be pointed out that the 12WF36 shape was chosen because its dimensions are such that it should have good resistance to local flange buckling. The test bears out this fact. The only case of flange crippling observed in the present frame before the failure occurred by the lateral buckling of the lee column was in the lower flange of the beam just at its intersection with the lee column. This buckle which took place in only one-half of the compression flange was observed by eye soon after ultimate load had been reached. The size of this wave did not increase in proportion to other deformations so that at the failure of the frame it was not much larger than when first observed. So far as could be detected this flange buckle did not affect the load carrying or energy absorbing capacity of the frame. There is the possibility that the damaging effect that this local buckle might have had was prevented or minimized by the presence of the lateral support strut just 4 inches from the center of the buckle wave.

In planning the present test, two methods of observing flange crippling were provided. A mechanical micrometer gage was mounted in such a way that the relative movement occurring between the flanges of the beam could be measured. These gages are denoted by the initials RFMD (Relative Flange Movement Dials) on Fig. 16 where their positions at the lee knee are shown. The

second method of detecting the flange crippling was the use of two SR-4 electrical strain gages mounted opposite one another on the inner and outer surfaces of the compression flange. Three such indicators were used in the region of the lee knee at the location shown on Fig. 16. The center of the visually observed flange buckle wave coincided with the location of SR-4 gages 35 and 36.

The relative flange movements at locations 1, 2, and 3 are plotted against the strain at the locations on Fig. 16. The strains are not determined with a high degree of accuracy but do show the order of strain magnitude when the lateral flange movements took place. The unit strain for RFMD -1 and -2 was computed from readings on the rotation indicator mounted between these two dials. The unit strains for RFMD -3 was taken from SR-4 electrical strain gages 35 and 36.

The rapid increase in relative flange movement shown by dials 1 and 2 at unit strains of about 20 x 10^{-3}in. per in. may be associated with the impending lateral buckle of the lee column which occurred at a strain of about 23 x 10^{-3}in. per in. since no local flange crippling was observing in the column flange. Displacements in the order of 700 x 10^{-4}in. were measured with RFMD -3, but these readings could not be plotted since the means of determining the unit strain was lost when the SR-4 strain gages stopped operating at a unit strain of about 20 x 10^{-3}in. per in.

Indication of flange crippling given by the SR-4 strain gages is shown on Fig. 17. Here the readings from pairs of SR-4 strain gages are compared, the individual gage reading being plotted against the average reading for the two gages at a particular section of the beam. SR-4 gages 35 and 36 were located

at the visually observed buckle and their comparison is of particular interest. The gage readings follow one another very well until a unit strain of about 11 x 10^{-3}in. per in. was reached where upon the two curves diverge. This would indicate flange crippling. The strain level at which the buckle was observed is marked Obs. F.B. (Observed Flange Buckle). The curves plotted from SR-4 gages 33 and 34 do not diverge and no flange buckling was observed even though the strain level was as high in their case as in the case of gages 35 and 36.

V. SUMMARY

The test frame, test apparatus and testing procedure has been described very briefly so that the test results could be interpreted. The details of the frame and test apparatus are shown on Fig. 1, 2, and 3. The system of loading proved to be a very good way of applying a combination of vertical and horizontal load.

The results of elastic and simple plastic analysis of the frame are given so that its behavior during test could be evaluated.

The major emphasis has been on the results of the tests. The following statements sum up the results of this frame test.

1. The elastic behavior of the frame was for all practical purposes identical to the theoretically predicted behavior when the increased flexibility of the knees was taken into account. Methods are available by which such elastic analysis of the knee may be made (see Ref. 1).

2. The analysis of data showed that the component parts of the frame behaved similar to separate isolated tests.

3. The ultimate load by test was 1.02 times greater than the collapse load predicted by simple plastic theory.

4. The actual deflection at predicted collapse load was very close to the predicted value given by a plastic hinge method.

5. The frame was able to carry the predicted collapse load through deflections twice as great as those which existed when the maximum load was first reached.

6. The frame showed the ability to absorb relatively large amounts of energy. It finally absorbed about 9 times as

much energy as it had when the elastic limit had been reached and about 3 times as much as when the ultimate load had been reached.

7. The knee used in the frame was capable of carrying more than the plastic moment for the beam section without showing any signs of failure even though the rotation of the knee became about 5 times greater than the rotation at yield moment and 2.7 times greater than the rotation at plastic moment of the beam section.

8. The 12WF36 section used in the frame showed an ability to withstand large rotations at moments which were close to the theoretical plastic moment. Plastic hinge action started when the actual moment was about 96 per cent of the theoretical plastic moment. The beam was able to undergo unit rotations in the order of 16 times greater than the theoretical unit rotation at the predicted yield moment (Fig. 10). This rotation took place without flange crippling in the regions where the rotation was measured.

9. The magnitude of the lateral support forces required to insure the good plastic action of the frame was relatively small. The largest force measured at a single support point was about 2 per cent of the resultant of compression stresses at plastic moment in the beam. The total of either the tensile or compressive lateral forces was not more than 7 per cent of this resultant.

10. The largest lateral forces were measured at the hinge locations.

11. The frame was subjected to lateral buckling when large regions became plastic. The adverse effects of this buckling was minimized by a stiff lateral support system. All signs

of plastic instability occurred after the ultimate load had been reached.

12. Final failure was brought about by lateral buckling of the lee column after the frame had supported virtually its ultimate load through deflections 230 per cent greater than those when ultimate load was first reached. The column had no lateral support except at its intersection with the beam and at its base.

13. The 12WF36 shape was intentionally chosen to minimize the effect of local buckling. One small wave of flange buckling was detected soon after the ultimate load had been reached, but it did not develop further. The lateral buckling action previously mentioned caused final failure.

In general, the results furnish encouraging evidence of the applicability of plastic analysis in structural design. The frame showed the characteristic behavior of structural elements and frames when loaded in the plastic range.

VI. ACKNOWLEDGMENTS

The test and resulting analysis described in this report is part of an investigation carried on as a result of a cooperative agreement between the Welding Research Council of the Engineering Foundation, the Navy Department, and the Institute of Research of Lehigh University. The investigation is one phase of the structural research program of Fritz Engineering Laboratory, of which Professor W. J. Eney is Director.

The test reported herein was carried out through the efforts of several persons in addition to the authors. Much of the early planning for the test was done by K. E. Knudsen, former Research Assistant Professor. J. P. Verschuren, former Research Assistant, designed parts of the test apparatus. In addition, the competent work of the laboratory machinists and technicians under the direction of K. R. Harpel, Foreman, helped bring the test to a successful conclusion.

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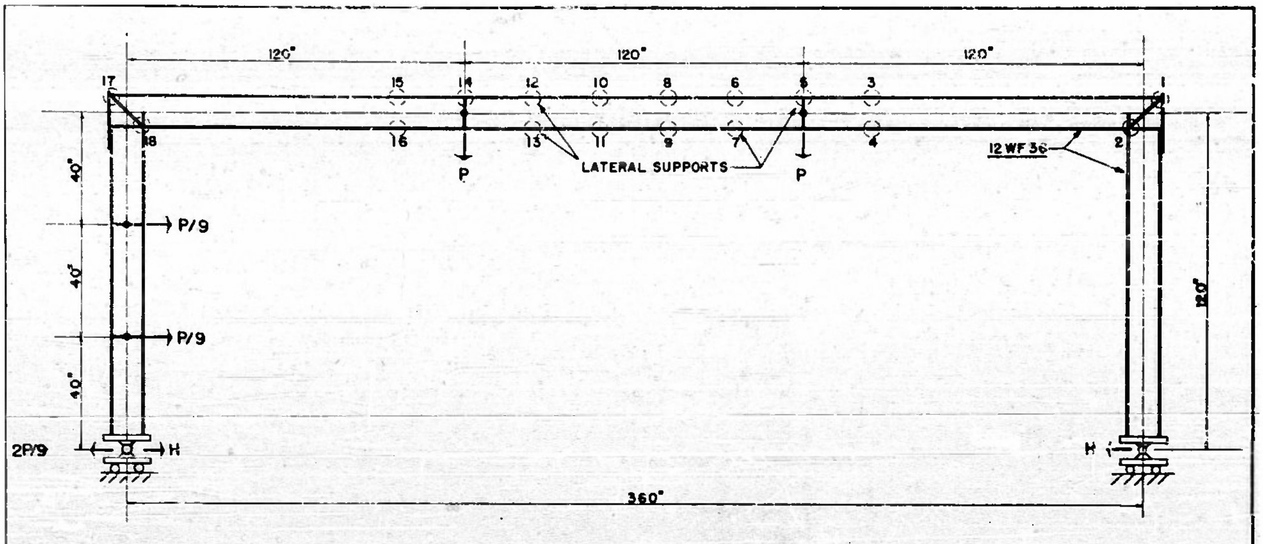


FIG. 1 FRAME DIMENSIONS AND DETAILS

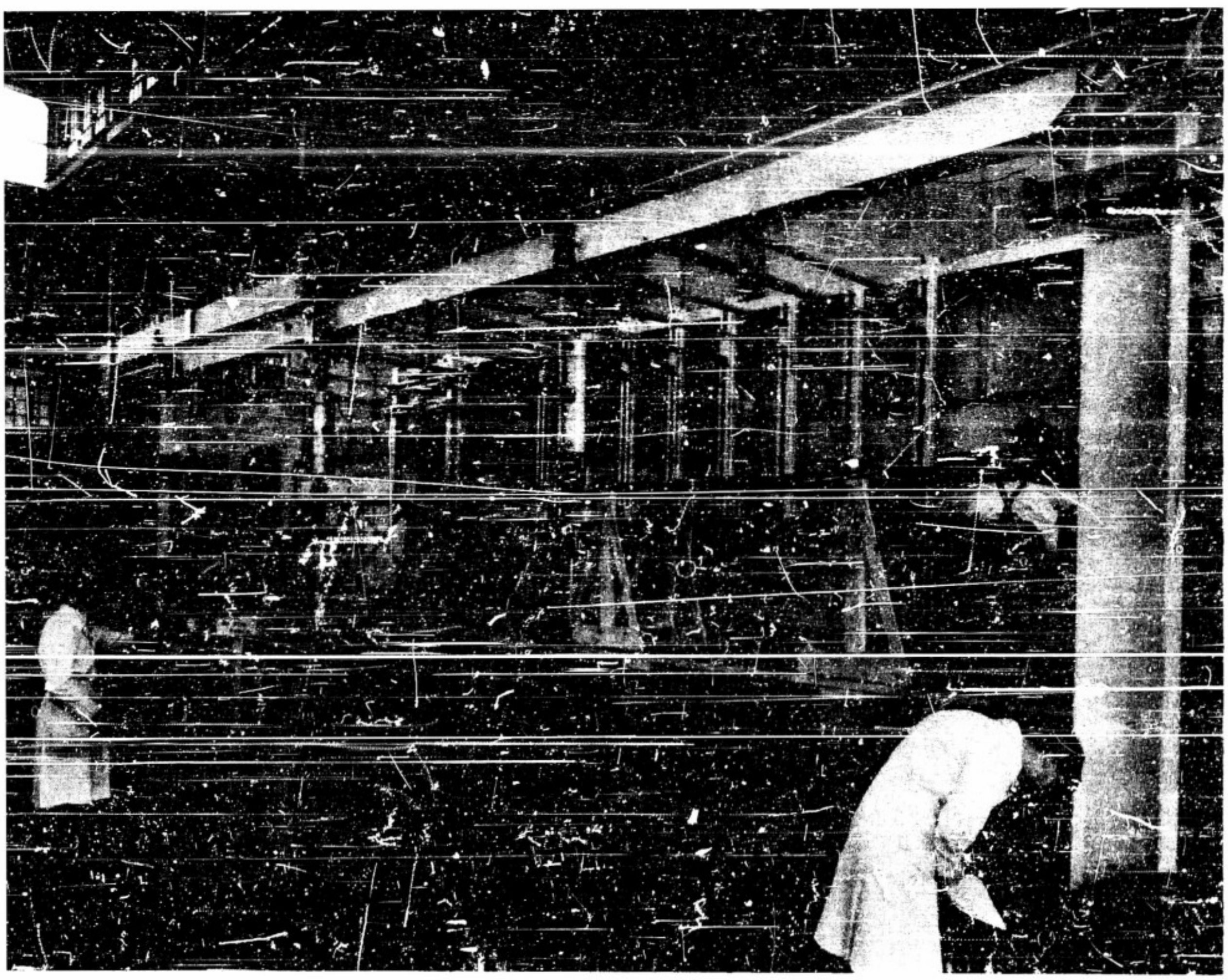
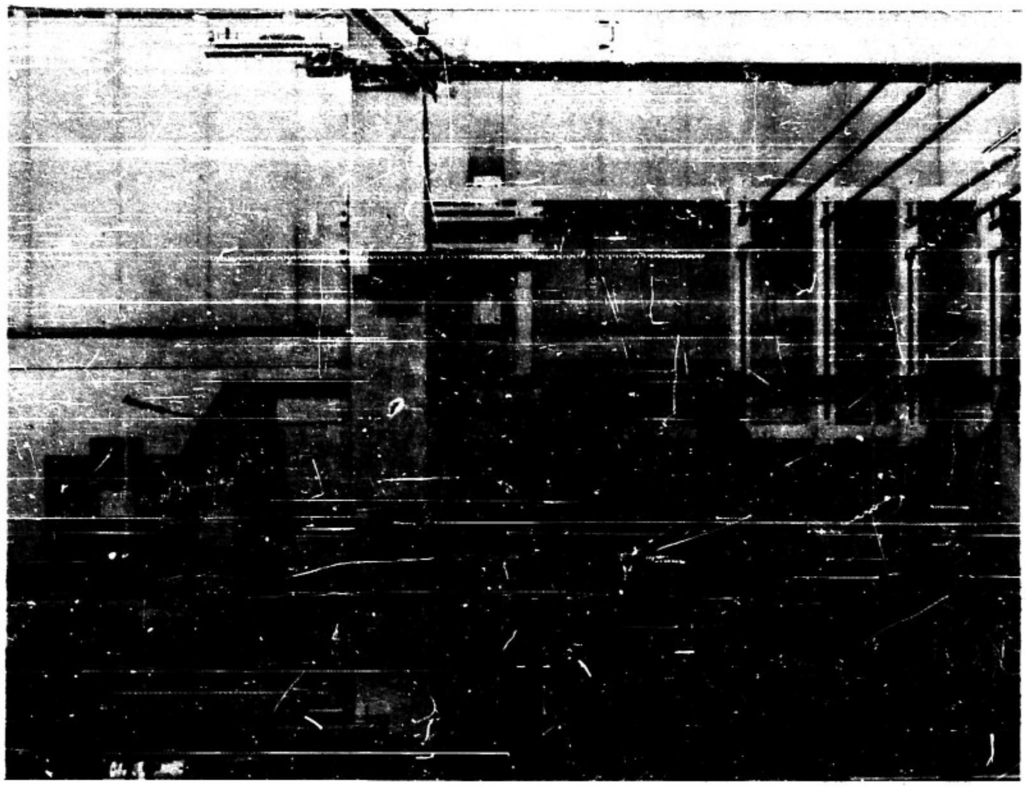
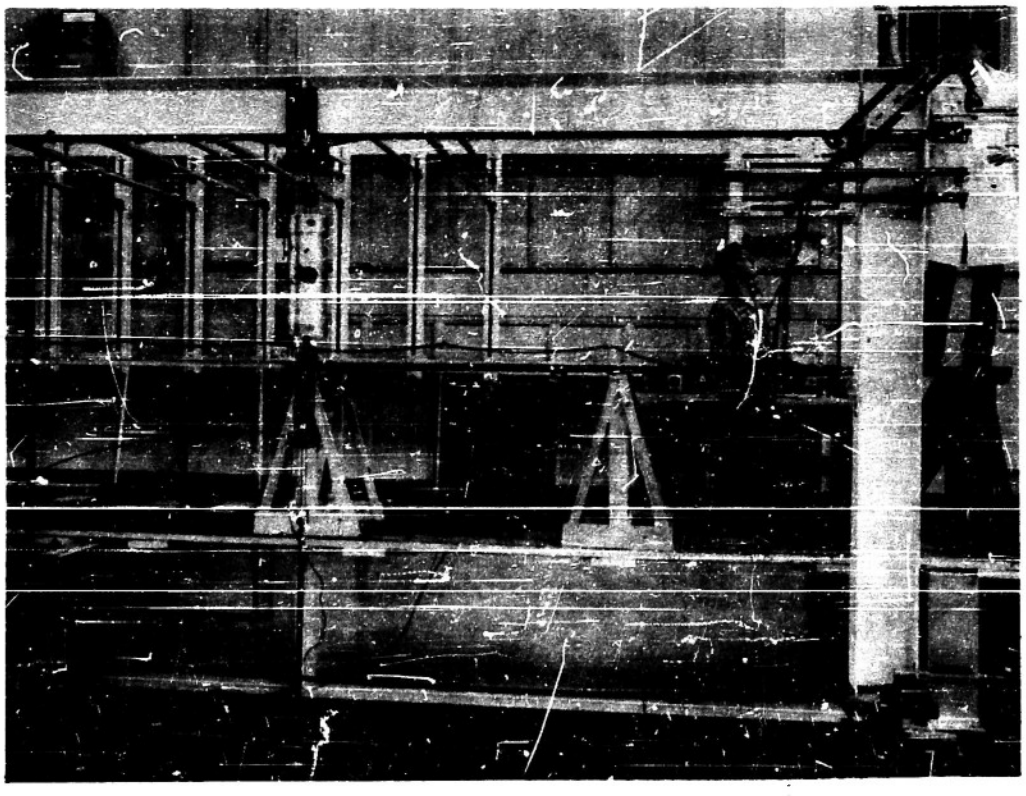


FIG. 2 GENERAL VIEW OF FRAME AND TEST APPARATUS



a. WINDWARD HALF OF FRAME



b. LEE HALF OF FRAME

FIG. 3 CLOSE-UP VIEW OF FRAME AND TEST APPARATUS

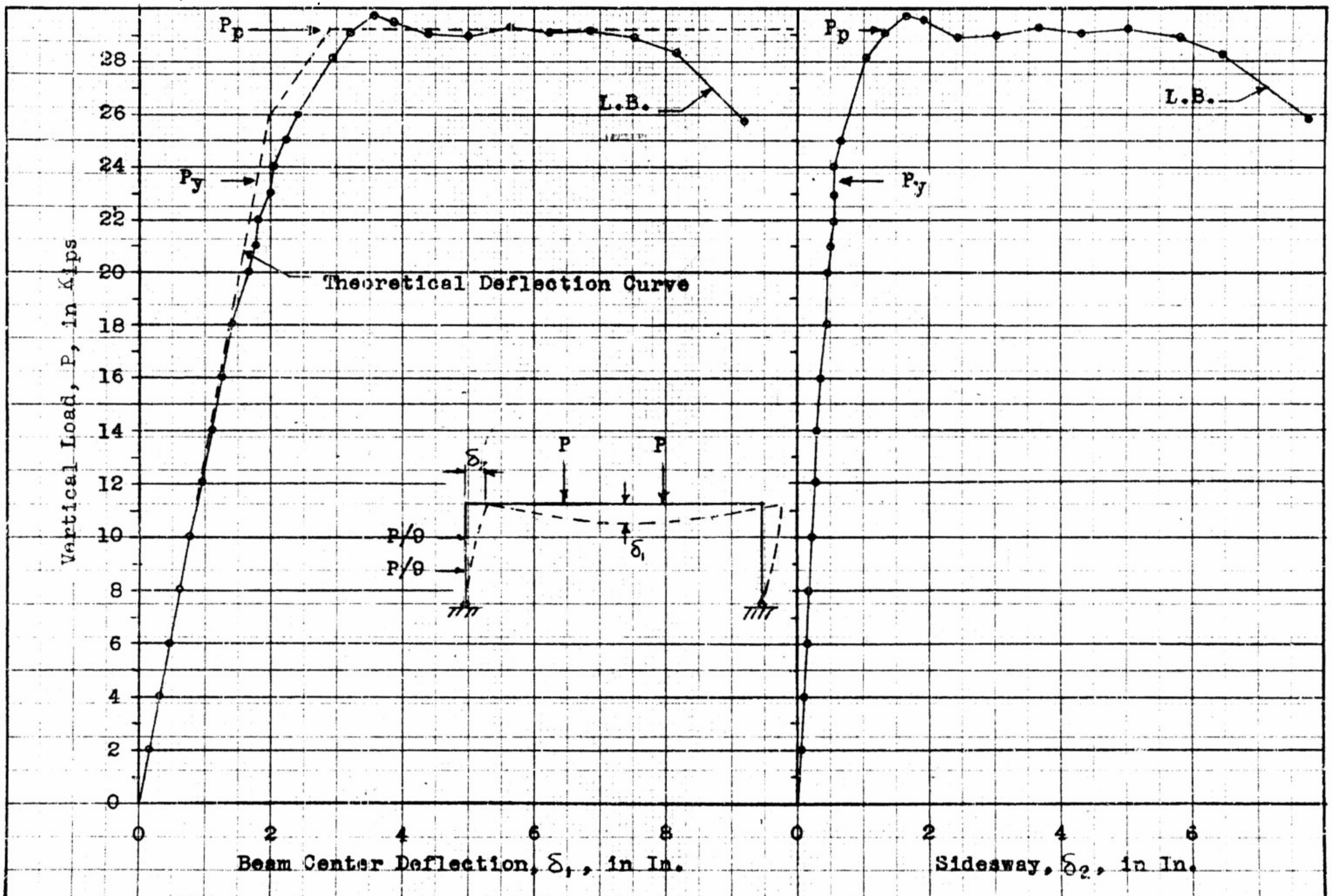


FIG. 4 LOAD-DEFLECTION CURVES

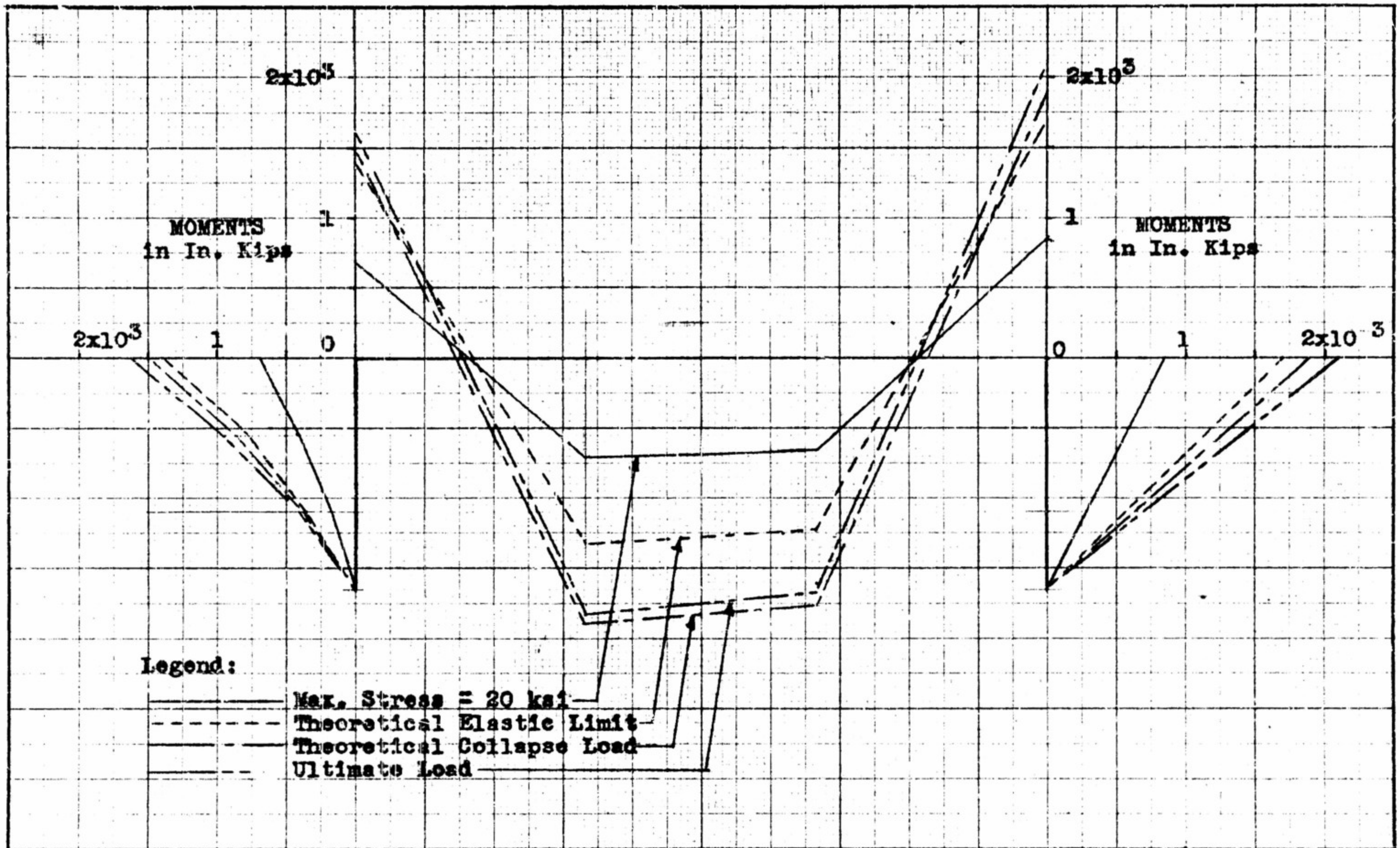


FIG. 5 FRAME MOMENT DIAGRAMS

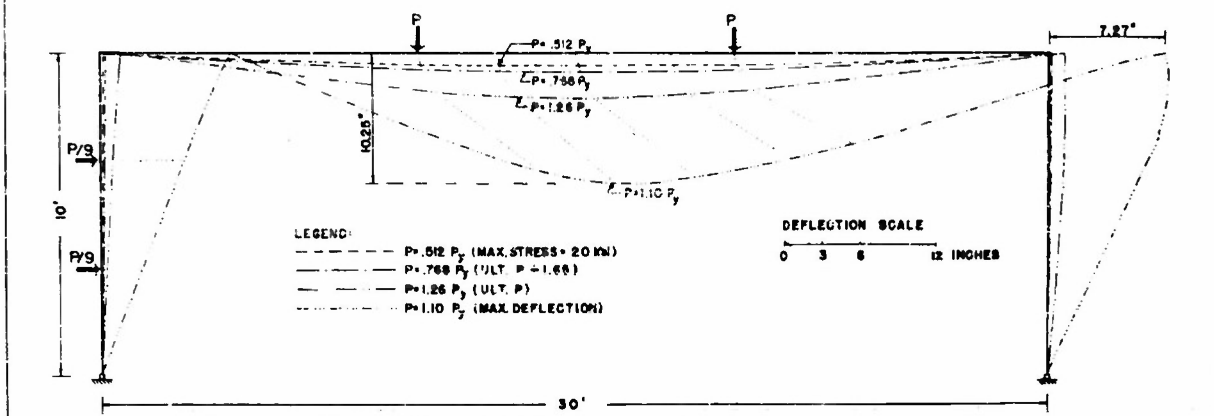


FIG. 6 FRAME DEFLECTION CURVES

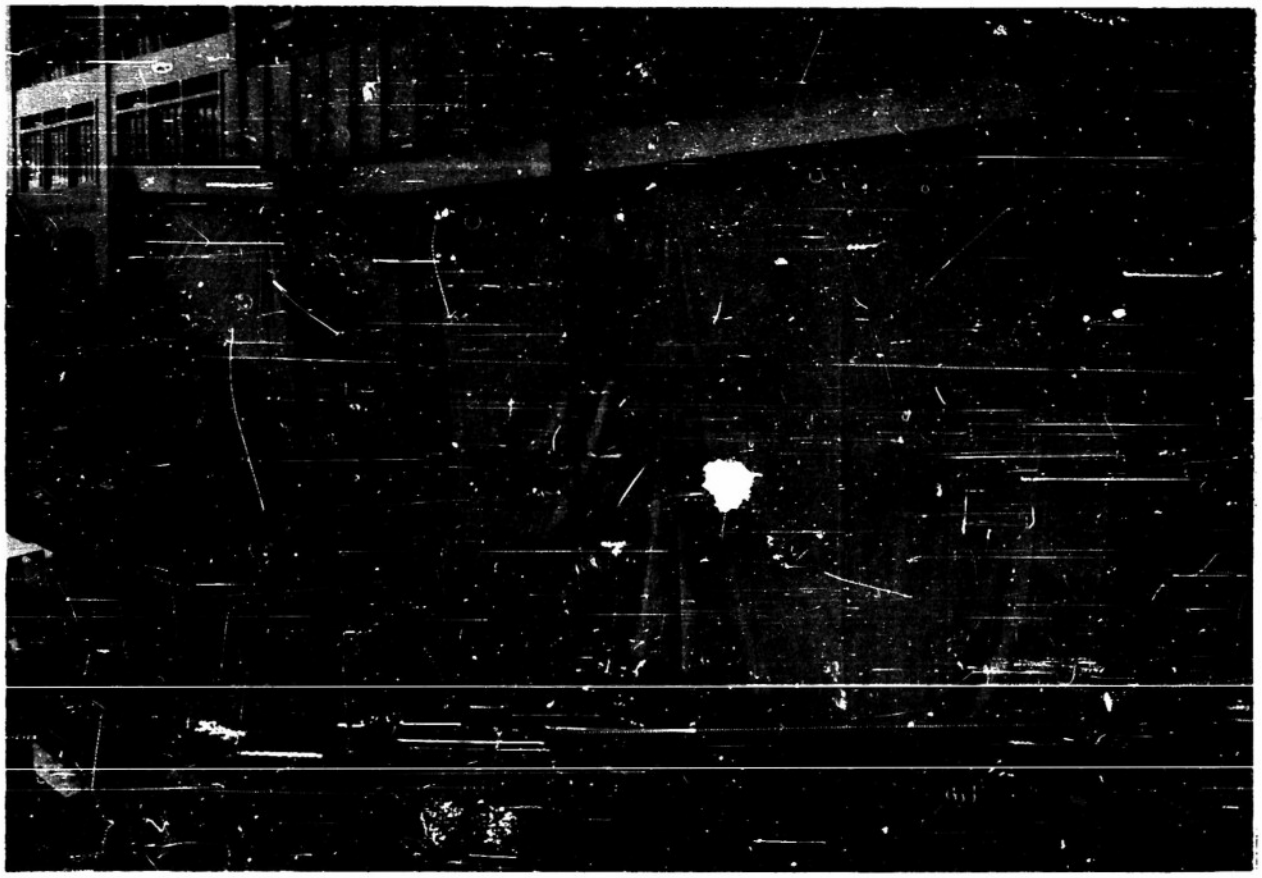


FIG. 7 GENERAL VIEW OF FRAME AFTER TESTING

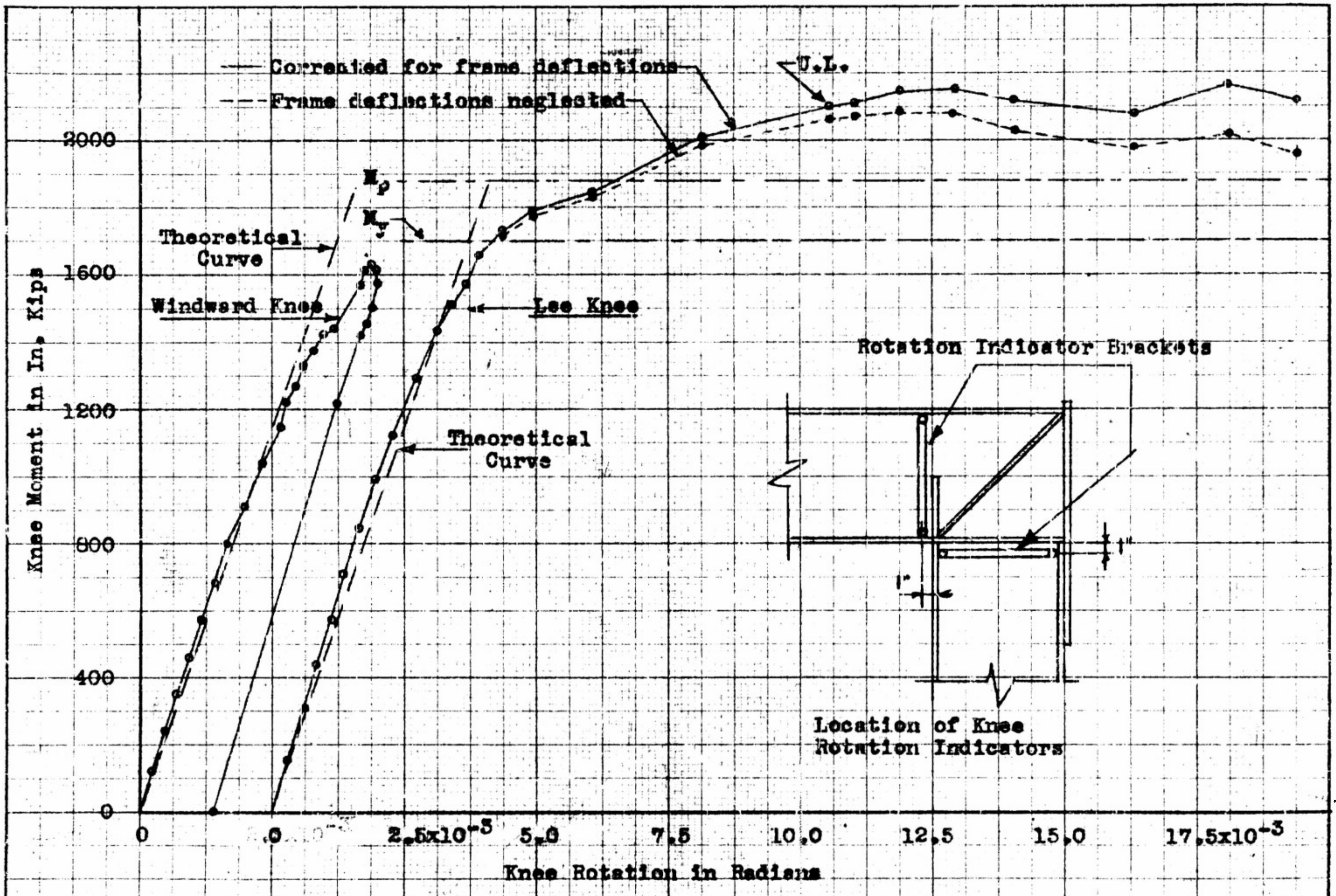
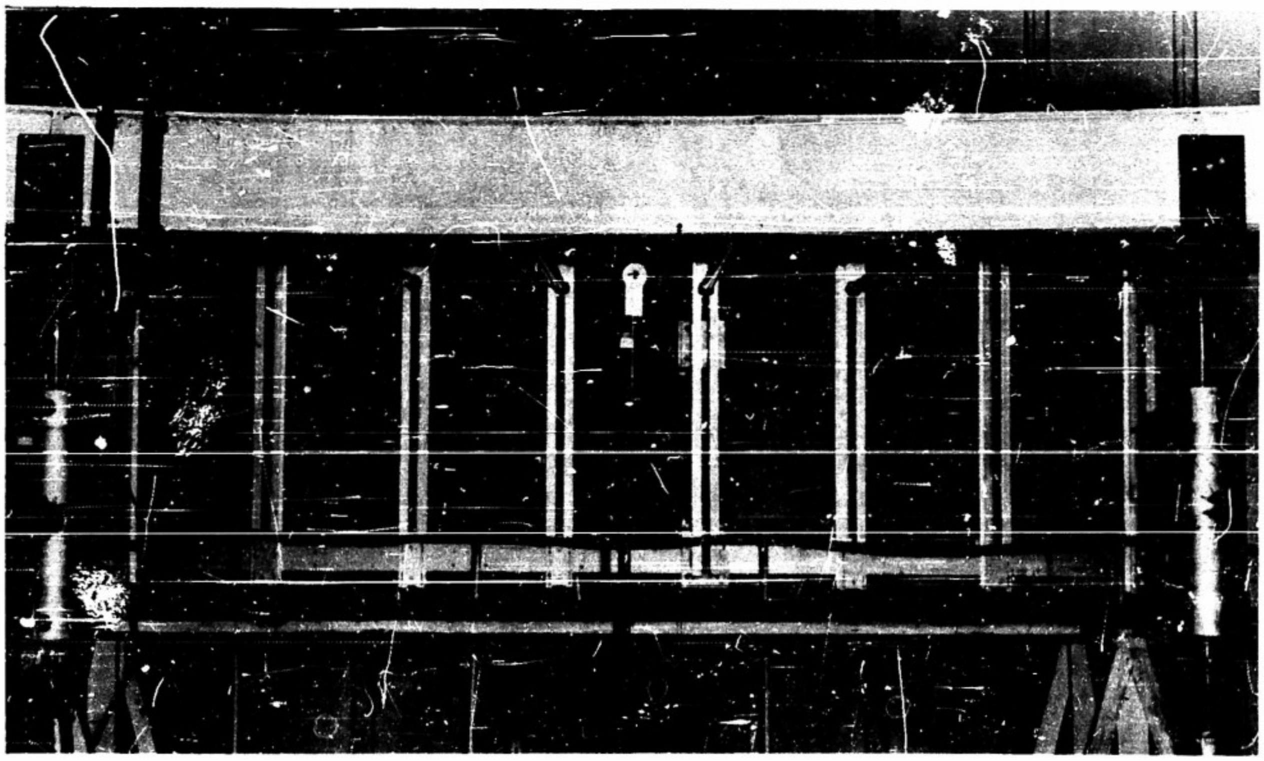


FIG. 8 MOMENT-ROTATION CURVES FOR KNEES

a. FIRST PLASTIC HINGE AT LEE KNEE



b. SECOND PLASTIC HINGE AT WINDWARD VERTICAL LOAD POINT

FIG. 9 ZONE OF YIELDING AFTER ULTIMATE LOAD

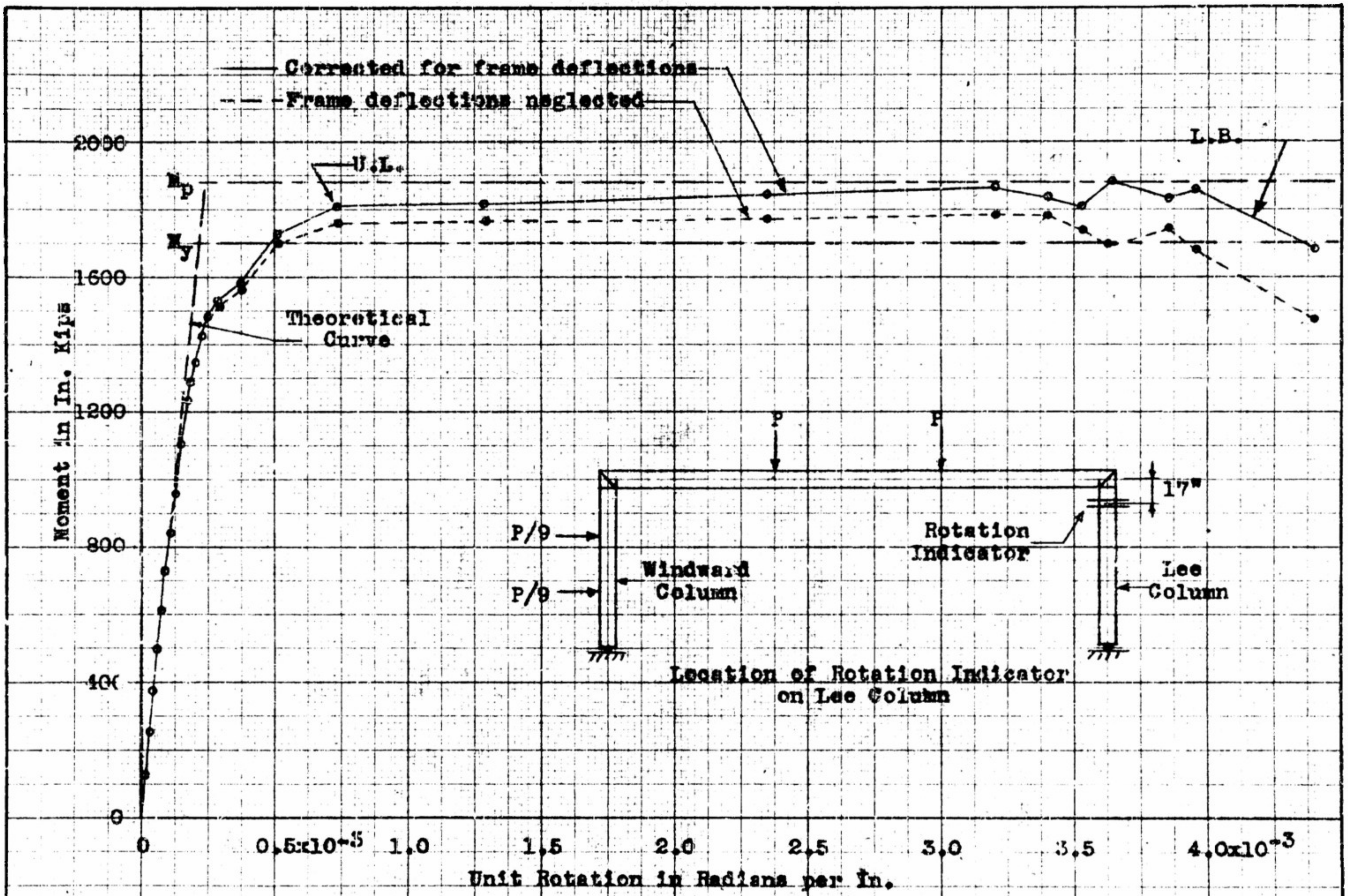


FIG. 10 MOMENT-UNIT ROTATION CURVE FOR LEE COLUMN

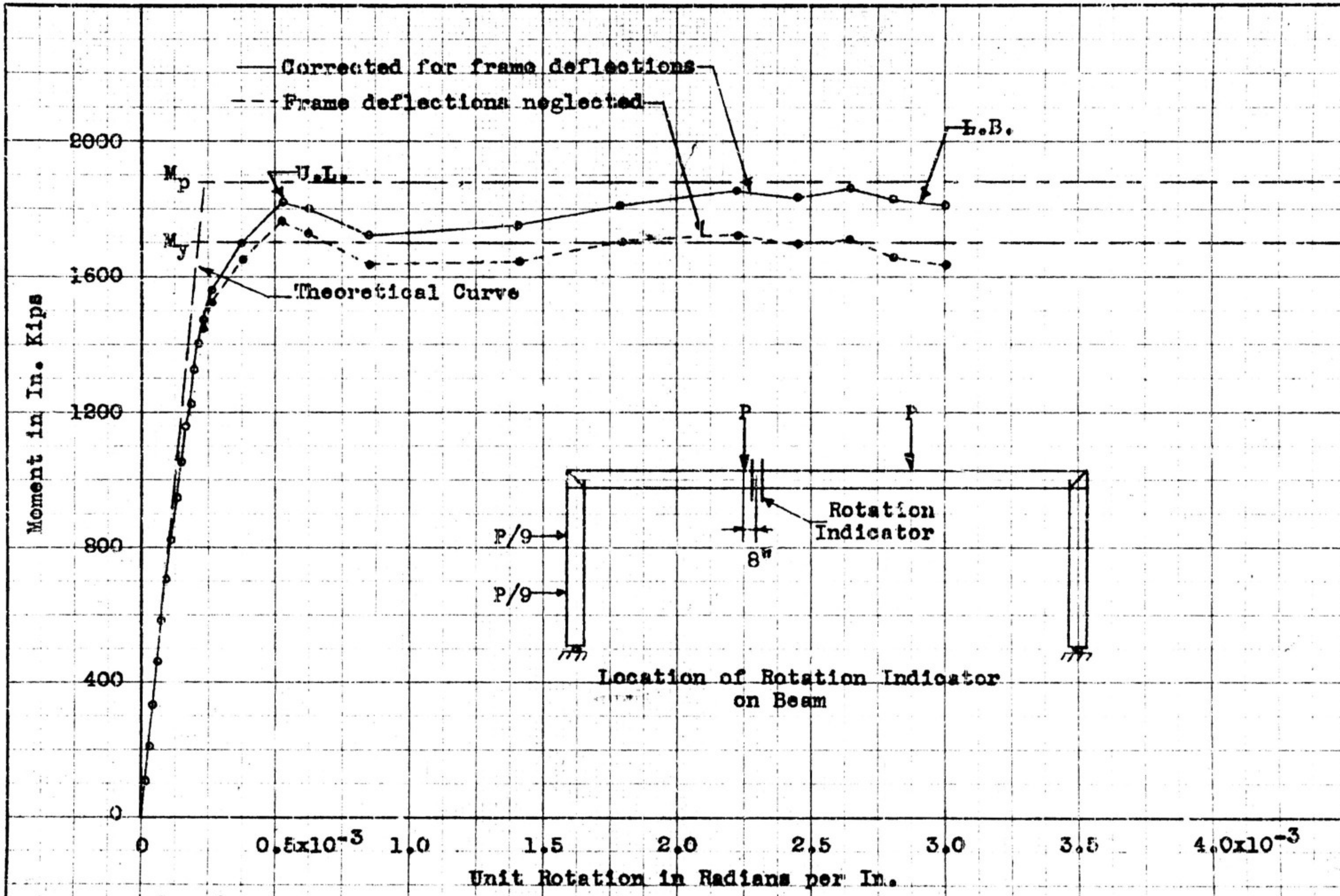
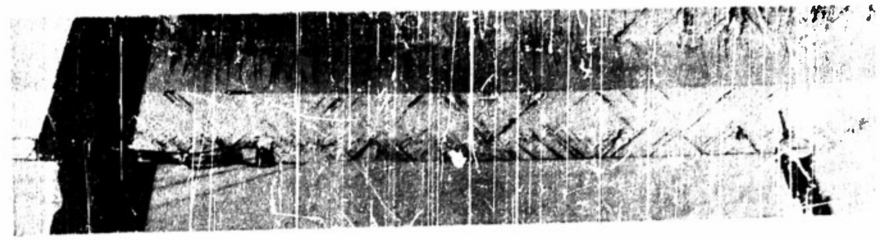


FIG. 11 MOMENT-UNIT ROTATION CURVE FOR BEAM

a, VIEW OF INSIDE FLANGE

b. SIDE VIEW

FIG. 12 LEE COLUMN AT FINAL LOAD



a. VIEW LOOKING UP

b. SIDE VIEW

FIG. 13 REGION OF SECOND PLASTIC HINGE AT FINAL LOAD

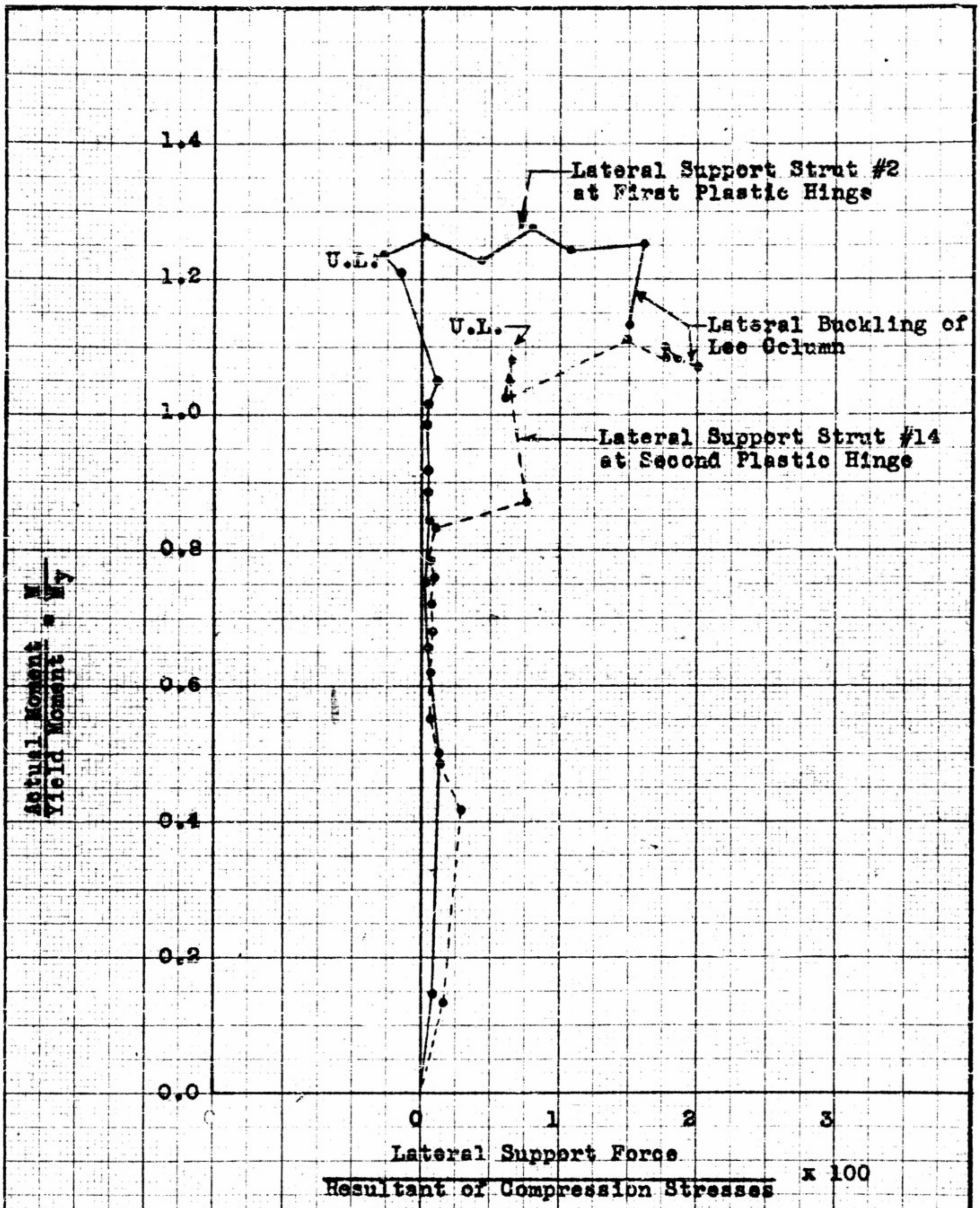


FIG. 14 MOMENT VS. LATERAL FORCE AT PLASTIC HINGES

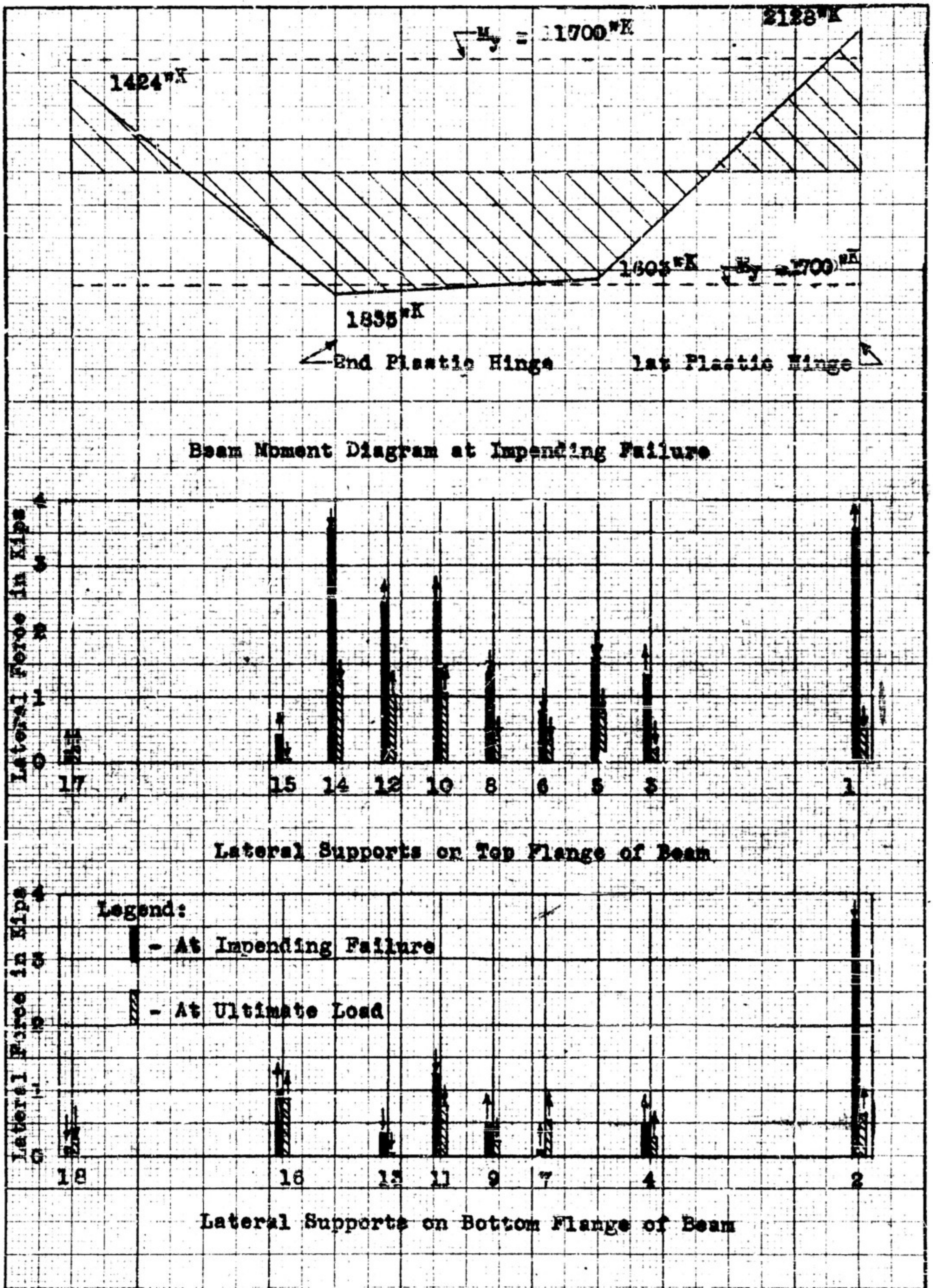


FIG. 15 DISTRIBUTION OF FORCES IN LATERAL SUPPORTS

NO. 340R-20 DIETZGEN GRAPH PAPER 20 X 20 PER INCH

EUGENE DIETZGEN CO.

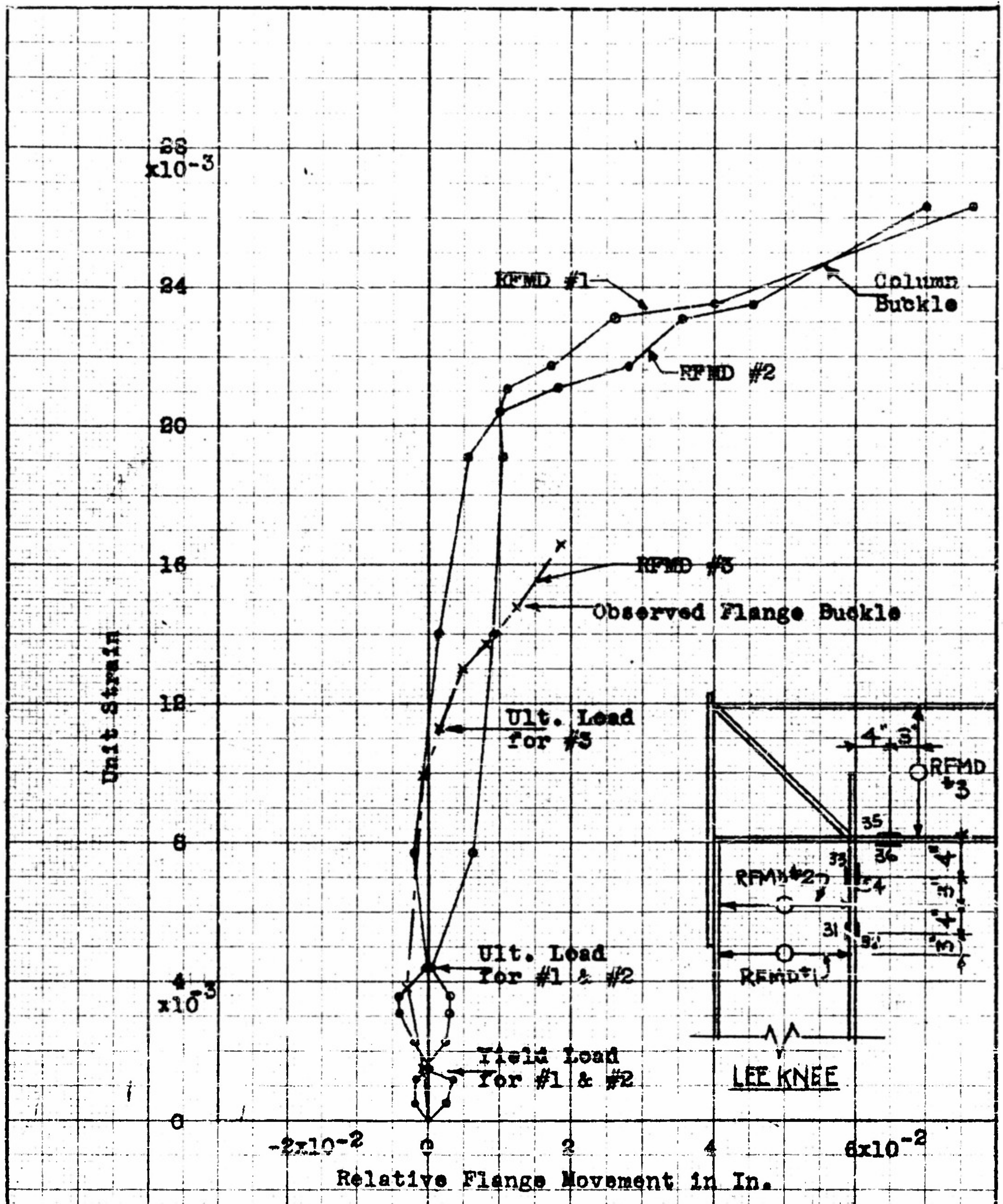


FIG. 16 RELATIVE MOVEMENTS BETWEEN FLANGES LEE KNEE

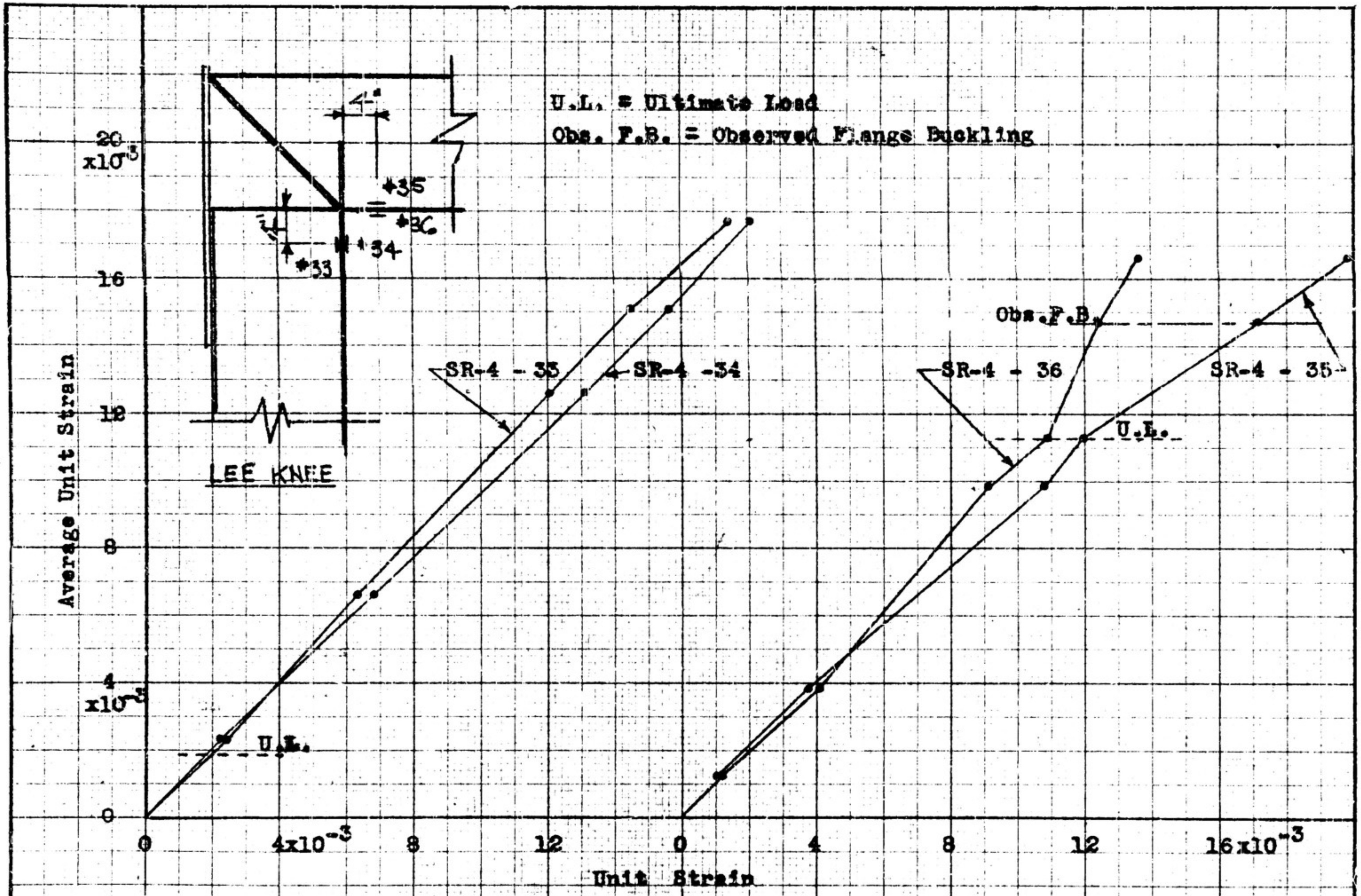


FIG. 17 FLANGE CRIPPLING AS SHOWN BY SR-4 STRAIN GAGES